

ARTICLE

Retrofitting Steel Moment Frames Using Cable Bracing

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ABSTRACT

In this paper, the behavior of retrofitted steel moment frames with bracing has been investigated. Braces include double-channel cross brace, cross braces with cable and brace with two cables passed through a cylindrical steel sheath at the location of the cables. Nonlinear analysis of frames has been carried out under cyclic loading with increasing amplitudes. Comparison of numerical analysis results with laboratory data shows the accuracy of finite element models. By determining the hysteresis and plasticity behavior of the frames, advantages and disadvantages of each of the retrofitting methods have been examined. The results have shown the use of double channels and cables to retrofit the frame increases the initial hardness and final load of the frame considerably compared to the moment frames and reduces its ductility. In frame with sheathed cable brace, the initial hardness was the same with the moment frames and the frame has been shown to have ductile behavior.

1. Introduction

Steel moment frames are designed in areas of high and very high seismic risk as well as medium to high ductility. These frames have sufficient ductility and amortization capacity with tolerance of great plastic deformities and rotations at the site of plastic hinges and fittings. Thus, the coefficient of behavior is considered to be large in the design of such frames and their design forces are less than other structural systems. Reinforcement of existing moment frames becomes necessary in cases such as unpredictable incidents, severe earthquakes or structural changes. Some researchers have tried to examine the retrofitting of moment frames using convergent and divergent braces^{[1][2][3][4][5]}. Other studies have been carried out to retrofit the steel frame using buckling restrained brace^{[6][7]} or a steel shear wall^[8]

^{[9][10]}. Some disadvantages are observed in conventional retrofitting methods for moment frames, by adding braces including increased axial force of pillars adjacent to braces because of the bracing operation and therefore the need for retrofitting the pillars and foundation, change of the ductile frame behavior to brittle behavior, buckling of the compressive member of brace and a permanent deformation are observed in the frame^{[11][12][13]}. In order to eliminate these disadvantages, some studies been done in which methods such as the use of non-pressure braces, buckling-resistant braces, and dissipating steel bracing have been examined in order to remove the possibility of buckling and maintain the frame ductility^[13]. The research seeks to find a way to simultaneously meet the goals outlined above. A method for retrofitting steel moment frames has been suggested using cable brace in which

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two steel cables are passed through a cylindrical sheath at a point of impact. The bracing member does not come into action for low to moderate amplitude of vibrations. It controls the relative displacement for large amplitude of vibrations among the floors in a given range. Also, the frame behavior with a proposed bracing, moment frame without brace and curved frames with cross cables under cyclic loading have been experimentally investigated. In the present paper, the behavior of steel moment frames retrofitted with three types of bracing has been studied. To retrofit steel moment frames, three ways including use of cross brace with double-channels, cross brace with cables as well as brackets are examined with two cables passed through a cylindrical steel sheath at the junction of the cables. Finite element models of single moment frame as well as three retrofitted moment frames have been created using the ABAQUS software^[14].

Nonlinear behavior of steel materials nonlinear contact among cable components, brace and steel sheaths as well as nonlinear geometric formulation have been considered to predict rotating cylindrical sheath in finite element models. Moreover, nonlinear analysis of frames has been carried out under cyclic loading with increasing amplitudes. Comparison of results obtained from finite element analysis has been done with increasing amplitudes. Comparing finite element results with experimental data shows the accuracy of finite element models. For the examined frames, Force-displacement cyclic behavior, plasticity in frames and failure mechanism of the frames have been determined and investigated.

Retrofitted frame with cables passed through steel sheath shows appropriate behavior against cyclic loading with increasing amplitudes. The results show that these frames have a ductile behavior like steel moment frames and their energy dissipation capacity is the same as the main frame capacity. In this method, due to the lack of possibility of buckling, persistent deformation is not created in the frame and ductility required for the members is not increased by limiting relative displacement between classes by cables. In this retrofitting method, the axial force of the columns is limited and as a result, there less need to retrofit columns and foundations.

2. The Moment Frame Geometry and Retrofitted Frames

Behavior of moment frame without being retrofitted has been compared with the behavior of frames reinforced by double channel cross brace, cross cable brace, and brace with two cables passed through a cylindrical sheath at the junction of cables. These frames have been named as MF, MF-B, MF-C1 and MF-C2 respectively.

Members of the beam and columns of frames, profile H-150×150×7×10 is of SN400B mild steel. The design of the frames has been done with this philosophy that no collapse or buckling in the column should not be created in the event of a bounded frame failure. Also, in case of bounded failure, the behavior of the beam and column should be maintained. The used cable is of 316SUS type with a 7×19 strings and a nominal diameter of 10 mm, a Yield strength of 57.9 and a final strength of 2.60.2 kN. The braces have been made from 2UNP10 profile. Figure 1 shows the geometric characteristics of the moment and braced frames. The models details are listed in the Table 1.

Table 1. Model Details

Model	Beam profile	Column profile	Brace profile	Cable diameter (mm)
MF	H-150×150×7×10	H-150×150×7×10	-	-
MF-B	H-150×150×7×10	H-150×150×7×10	2UNP100	-
MF-C1	H-150×150×7×10	H-150×150×7×10	-	10
MF-C2	H-150×150×7×10	H-150×150×7×10	-	10

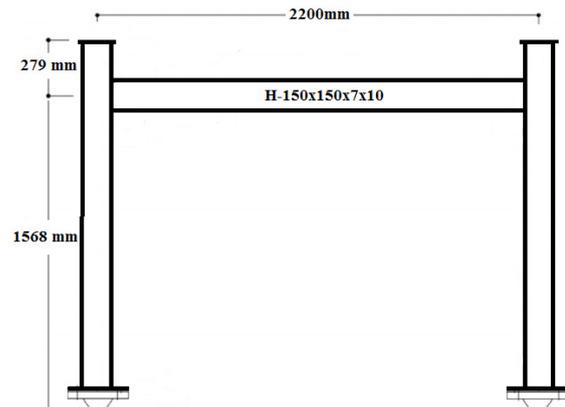


Figure 1. Geometric Characteristics of the Moment Frame

In a sample with cable brace and cylindrical steel sheath, specifications of the sheath include a length of 214, an internal diameter of 40 and a wall thickness of 15 mm. To prevent tension concentration and damage to the cable, the inside edge of the sheath has been rounded to a radius of 5 mm. Figure 2 indicates the finite element model of steel frame with cable brace. In these models, cyclic loading up to a maximum displacement of 300 mm has been used.

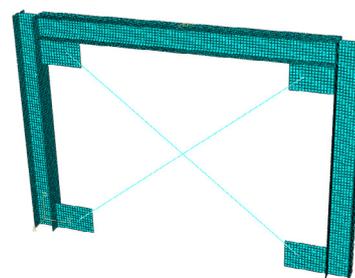


Figure 2. Finite Element Model of Moment Frame with Cable Brace

The deformation of the braced frame with the cable passed through the steel sheath is shown in Fig. 3 after exerting the lateral forces. In this frame, the cable length is greater than the diameter of the frame. Under the influence of the Q load, the frame begins to change. From the early stages of loading, tension occurs on all four cables when the member of the AB directly becomes straight ($\delta = \delta_s$). The cable shows a considerable resistance to the side movement. δ_s is determined due to the relative permitted displacement of the floor in the retrofitted frame and expected ductility from the moment frame. The relative displacement δ_s , in which the braces start to work, is calculated based on the length and diameter of the cylinder, the dimensions of the frame and allowable relative displacement between floors.

If the length of the cable inside the sheath is d_p , cable length out of sheath is L_B , the height of the frame and its length are h_c and h_b , δ_s is the lateral displacement created in the lateral frame deformation from the initial state to another state in which the frame diameter reaches the $2L_B + d_p$. By adjusting the L_B , the diameter and length of the sheath, the displacement can be limited by the permissible lateral change δ_s .

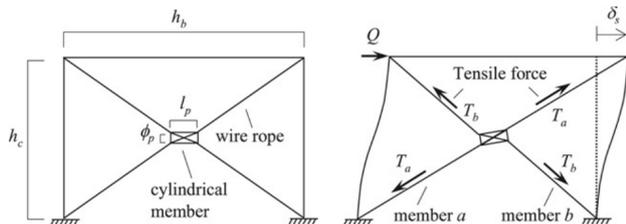


Figure 3. The Proposed System for the State with Cylindrical Member

3. Finite Element Model

The element used for the beam, column, and connector components is the reduced S4R hexagonal type with 4 knots and a linear order. Cable with a reduced element of type B31 has been modeled with two groups and a cross-section area equal to the effective cross-section of the cable.

In finite element model, nonlinear geometric behavior of materials and large deformations have been considered. For modeling of steel hardening

Isotropic and Kinematic compound has been implemented which shows a more accurate behavior in seismic loading^{[15][16][17][18][19][20][21][22]}.

Table 2 shows the specifications of consumed materials. Young's modulus of cylindrical sheath of steel has been introduced to be 1500 times the modulus of the mild steel in order to remove its deformation.

Table 2. Specifications of Consumed Materials

Steel	E (GPa)	F _y (MPa)	F _u (MPa)	Ultimate strain
SN400	180	2810	3310	0.2
SS400	180	2530	3330	0.19
A490	210	8020	9620	0.007

To prevent stress concentration at the location of cable contact with the sheath wall, the inner edge of the sheath is rounded to a radius of 5 mm. The collision in the beam-to-column connection has been modeled as a hard-frictional fitting with a detachment feature after unloading. For modeling the pre-stress force, 0.55 of tensile strength of the screw has been applied by using the thermal properties of materials with the local temperature reduction of the screw trunk^{[15][16][17][18][19]}.

At the cable connection point to the brace plate, the obtained concentration of tension and numerical instability hinder the progress of analysis. To avoid tension concentration, 12 rotational joint wire connectors were used for each cable at the cable connection to the latches^{[19][20][21][22]}. The loading of finite element models is considered as a cyclic lateral displacement with increasing amplitude. The appropriate size of the elements has been determined by analyzing the convergence of each member. The loading pattern is presented in Figure 4.

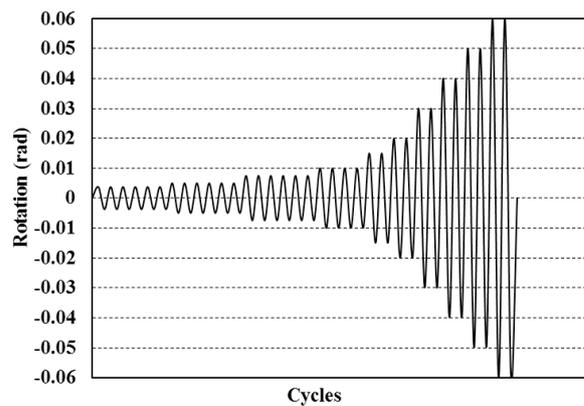


Figure 4. Loading Pattern

In Figure 5, curves resulted from the experimental and numerical work for the moment frame model of the steel with the sheath are shown. As it can be seen in the figure, loops of behavioral hysteresis curves in two finite element and laboratory modes are very close together in both methods and has a good compliance. As it can be seen in 50 mm lateral displacement, the descending stage of force-displacement curve is captured in the experimental model but there is no sign of reduction in finite element model. This is because of that experimental specimen failed in 55 mm lateral displacement and in the finite element model it is not considered.

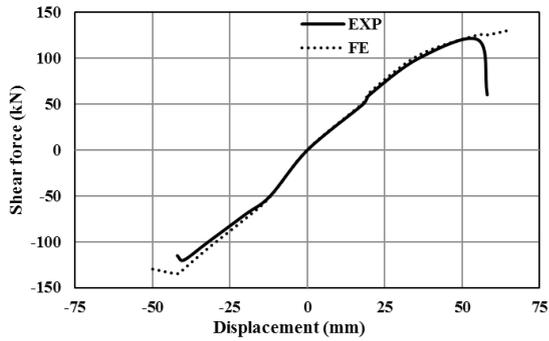


Figure 5a. Comparison of Experimental and Numerical Results – A Force-Displacement Graph

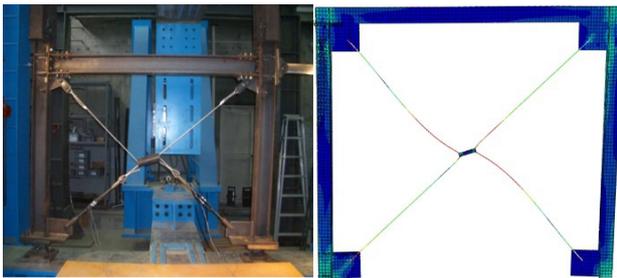


Figure 5b. Comparison of Experimental and Numerical Results – Deformation

4. Analysis of the Results

After analyzing the finite elements of these two models in Abaqus software, the obtained results will be observed in the visualization section. Therefore, the required results have been extracted and compared.

4.1 Plastic Strain and Stress Contour

In Figure 6, the contour related to the distribution of tension is shown in a variety of models. As it is seen in Figure 6a, in a conventional moment frame, tension concentration is situated at the location of column support and in the area of the beam connection to the column. The brace buckling occurs due to the compressive force in a moment frame with a conventional brace (Fig. 6b). As can be seen in Figure 6b, the stress concentration is situated in the location of column support and in the connecting area of the brace to the beam and column. Considering that buckling and large deformations occur in structural member of conventional brace during the first loading process and stiffness and resistance of the element is removed, it is impossible to use this member under pressure load. Due to the extreme tensile strength in a moment frame with a cable brace, one of the cables which is under the influence of tension and other one which is under pressure is left unused (Fig. 6c). Due to the presence of sheath in a moment frame with a sheathed cable brace, all frames and cables contribute to unloading

and it is expected that the cyclic behavior of the frame is symmetric (Fig. 6d).

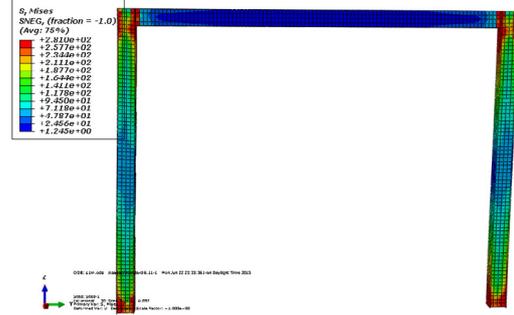


Figure 6a. Stress Contour of a Conventional Moment Frame

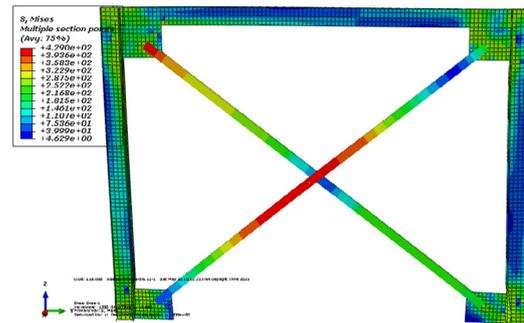


Figure 6b. Stress Contour of a Frame with Conventional Brace

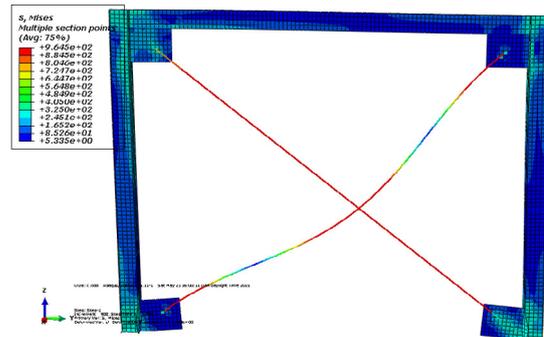


Figure 6c. Stress Contour of a Frame with Cable Brace

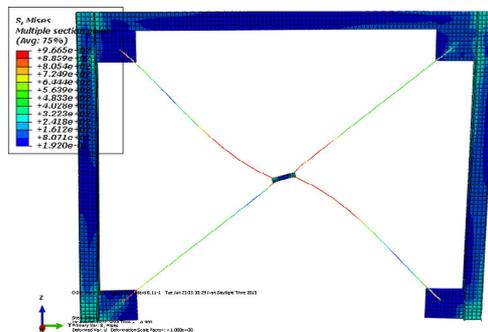


Figure 6d. Stress Contour of a Frame with Sheathed Cable Brace

In Figure 7, the contour related to the distribution of plasticity is shown in a variety of models. As it is shown in Figure 7a, the focus of plasticity in the conventional moment frame is situated at the location of column support and in the area of beam-to-column connection. Due to the compressive force in the moment frame with conventional brace, buckling is made in the brace (Fig. 7b). As it can be seen in Figure 7b, focus of plasticity is on the location of column support and in the area of beam-to-column connection.

According to the extreme tensile strength in moment frame with cable brace, one of the cables which is under the influence of tension effect and other one which is under pressure is left unused and the major plasticity occurs at the linking point of cable to connecting plate of beam to the column (Fig. 7c). All the frames and cables have shares in each cycle of loading in a moment frame with a cable brace due to the sheath and plasticity occurs mainly in the sheath. Moreover, the rest of the structural components including the beam and the column are in the elastic state (Fig. 7d).

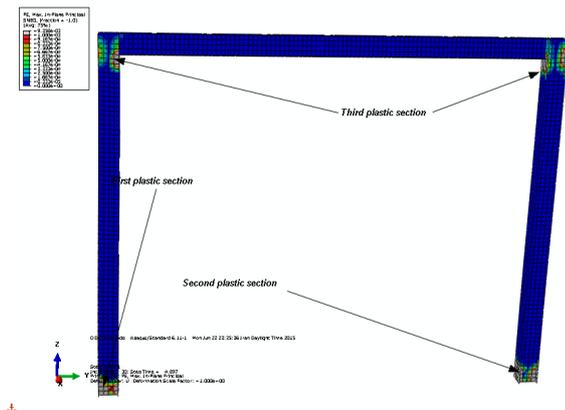


Figure 7a. Plasticity of a Conventional Moment Frame

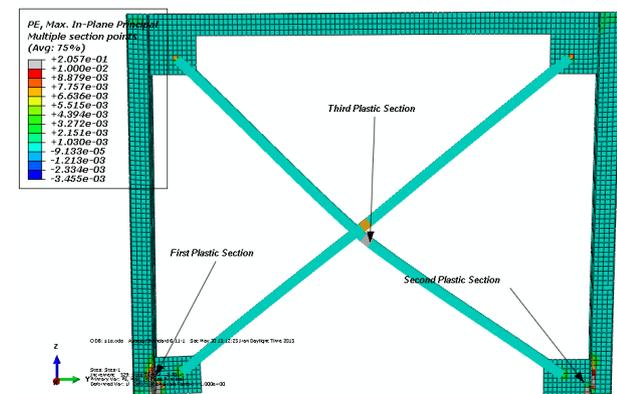


Figure 7b. Plasticity of a Frame with Conventional Brace

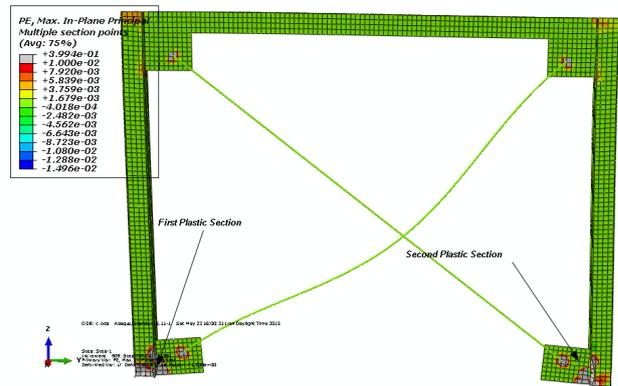


Figure 7c. Plasticity of a Frame with Cable Brace

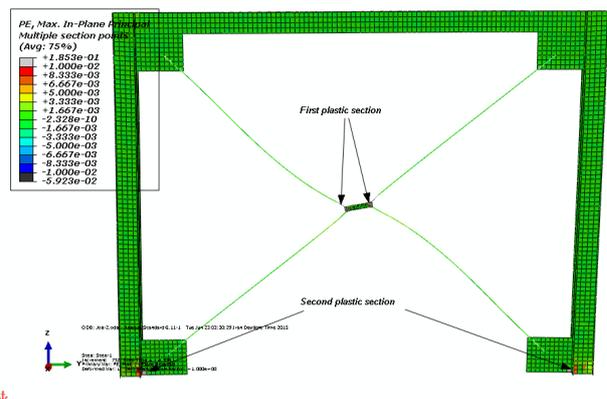


Figure 7d. Plasticity of a Frame with Sheathed Cable Brace

4.2 Hysteresis Graph

Figure 8a shows the force-displacement diagram in the moment frame under cyclic loading. As it can be observed, the hardness in the moment frame is very low. Stiffness is made due to the fact that lateral element loading is only the moment system of the frame. As it is shown in Figure 9a, this frame can withstand a maximum force of 211 kN.

According to Figure 8b, frame is associated with intangible losses after 30mm displacement in a moment frame with a cross brace based on Figure 8b, the mentioned frame can withstand 1250 kN in all cycles.

This means that the conventional brace shows uniform behavior in successive cycles, but the cycles have a small area. As it can be observed in Figure 8c, the frame faces a rupture in a moment frame with a conventional cable brace after 75 mm displacement. According to Figure 8c, this frame tolerates a maximum force of 1050 kN in the final cycle which means the cable brace has a high initial hardness, but its rupture occurs suddenly.

As it is seen in Figure 8d, after displacement of 30 mm in the moment frame with cross brace, the frame is accompanied by intangible losses. In Figure 8, the force-

displacement diagram of the moment frame with the sheathed cable brace under cyclic loading is demonstrated.

According to Figure 8d, this frame tolerates a maximum force of 1050 kN in all cycles. This means a conventional brace shows the uniform behavior successive cycles. Comparison of the graphs shown in Fig. 8 indicates that curve cycles in model of moment Frames with sheathed cable brace is more obese compared to other models.

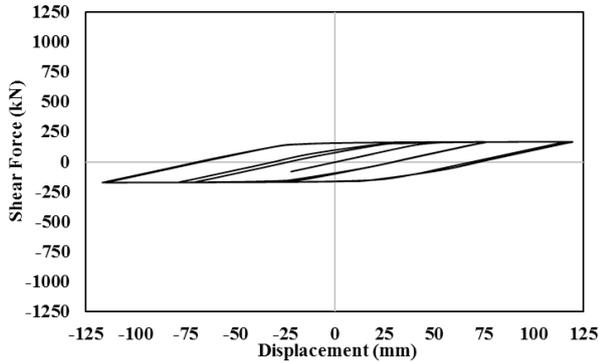


Figure 8a. Force-displacement Graph of a Conventional Moment Frame

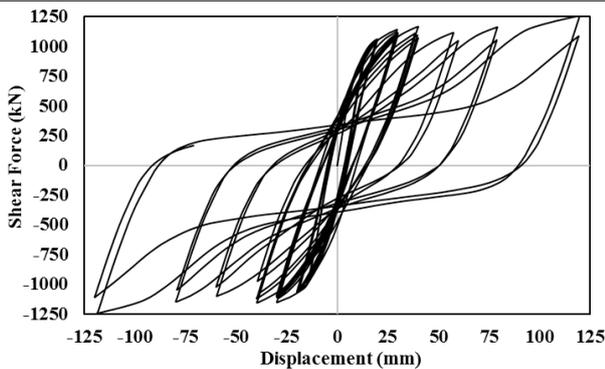


Figure 8b. Force-displacement Graph of a Frame with Conventional Brace

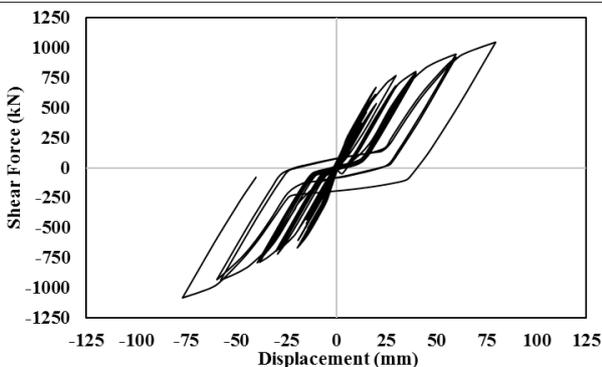


Figure 8c. Force-displacement Graph of a Frame with Cable Brace

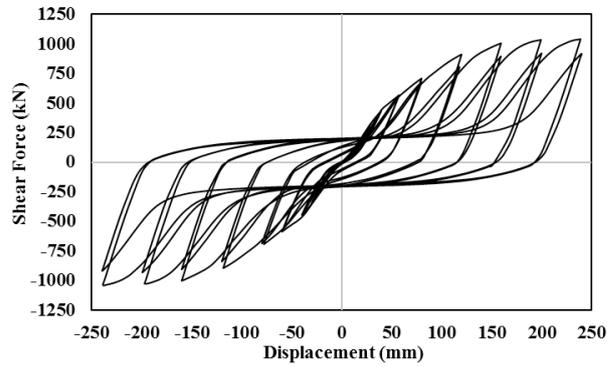


Figure 8d. Force-displacement Graph of a Frame with Sheathed Cable Brace

5. Dissipated Energy

Figure 9 shows the energy dissipated for a variety of models. As it can be seen, using sheathed cable brace increases the dissipation energy. The results of Figure 9 shows that braced frame and frame with sheathed cable brace dissipated energy more than other models.

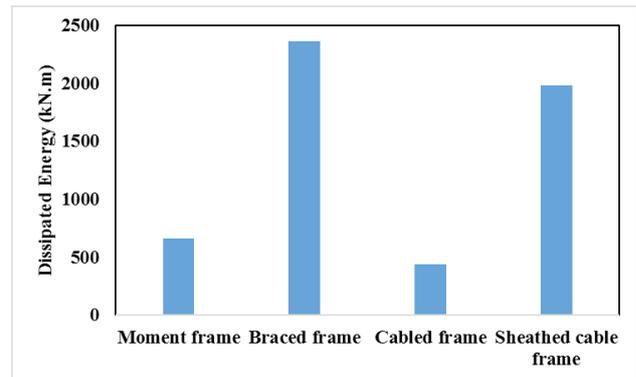


Figure 9. Dissipated Energy of Models

6. Conclusion

Finite element models of moment frame only and moment frame retrofitted with cable brace passed through a cylindrical steel sheath, cross brace with angles and cross brace with cable were created with the help of ABQUS software.

Nonlinear analysis of frames under cyclic loading has been done with increasing ranges. Comparison of numerical analysis results and experimental outcomes shows the accuracy of finite element models.

Basic cut, plasticity, cyclic force-displacement behavior and exact tension concentration have been precisely determined and reviewed.

The use of double channels for reinforcing steel moment frame leads to significant increase in the initial hardness and final load of the frame and results in decline in the final displacement. Reduced ductility and

energy dissipation capacity indicates that the permanent deformation is created in the frame due to the buckling of the compressive member in this type of retrofitting. In the frame reinforced with cross brace, the cables work from the early stages of loading to the tension and increase the initial stiffness of the frame in comparison with the hardness of the moment frame. In this case, the final load increases and the failure displacement decreases. In the mentioned frame, the cable brace transforms the frame's behavior from a ductile behavior to brittle behavior. Moreover, the results show that using sheathed cable brace increases the dissipation energy.

Narrow and unstable hysteresis cycles demonstrate the small capacity of this frame to resist lateral forces. In this method of reinforcement, retrofitting of columns and foundation is remarkable. The initial stiffness of the frame is the same for both the sheathed-cable brace and the moment frame. Force displacement diagrams of the two mentioned frames are consistent until the displacement of the δ_s . The cable does not affect the moment frame behavior up to the noted amount of displacement and straightening brace. Then, in the event of an invasion of forces from lateral forces such as δ_s , brace contribute to the frame's behavior with delay. Consequently, the behavior of the frame is a form of ductile behavior of moment frame up to displacement of δ_s . By adding brace to these frames, resistance of the frame is increased while its ductility is maintained. In this retrofitting method, due to the lack of possibility of buckling, persistent deformation cannot be made. In addition, required ductility for the members is not increased by limiting relative displacement between floors using cables.

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